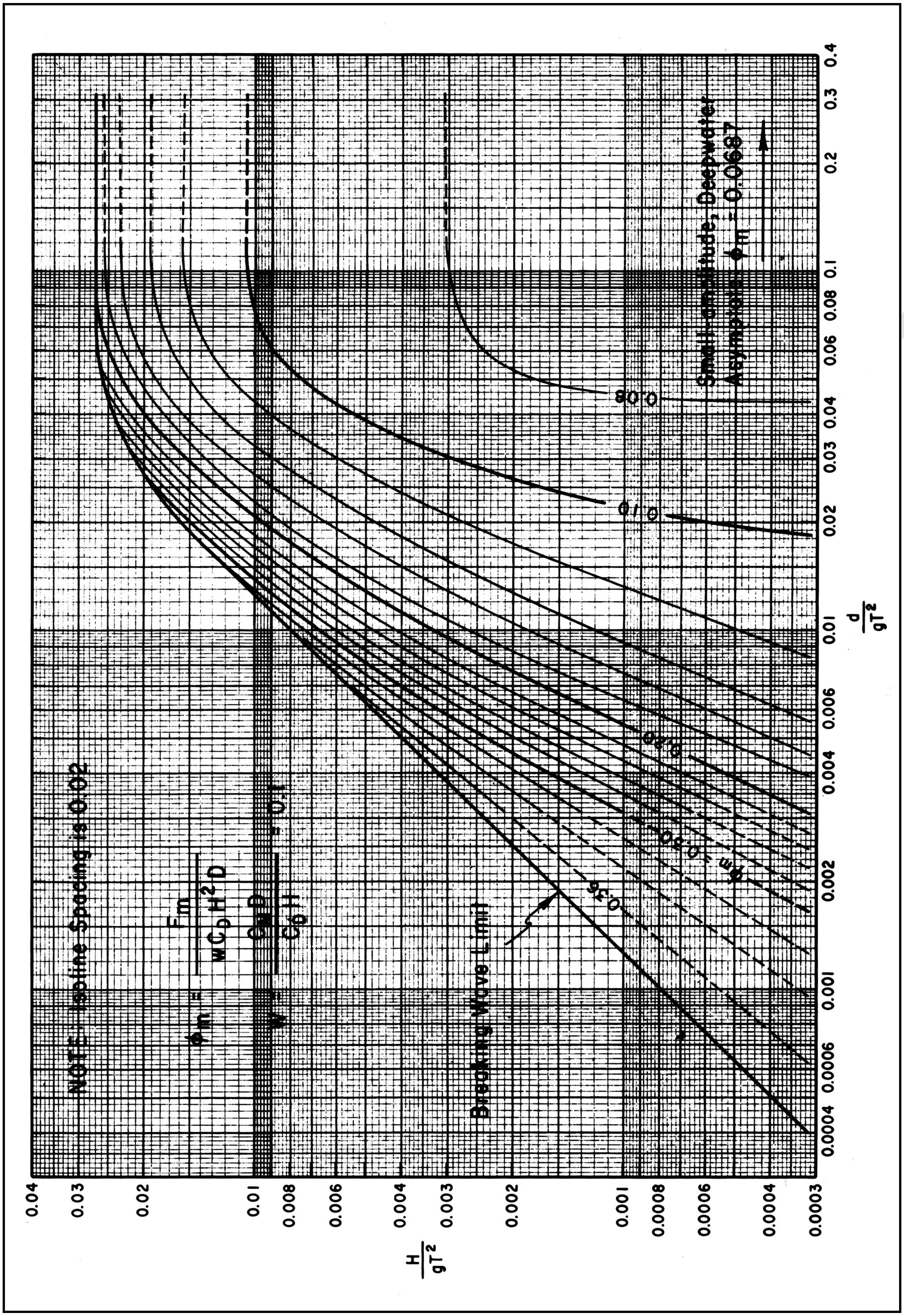

Figure VI-5-132. Isolines of φ_m versus H/gT^2 and d/gT^2 (W = 0.10)

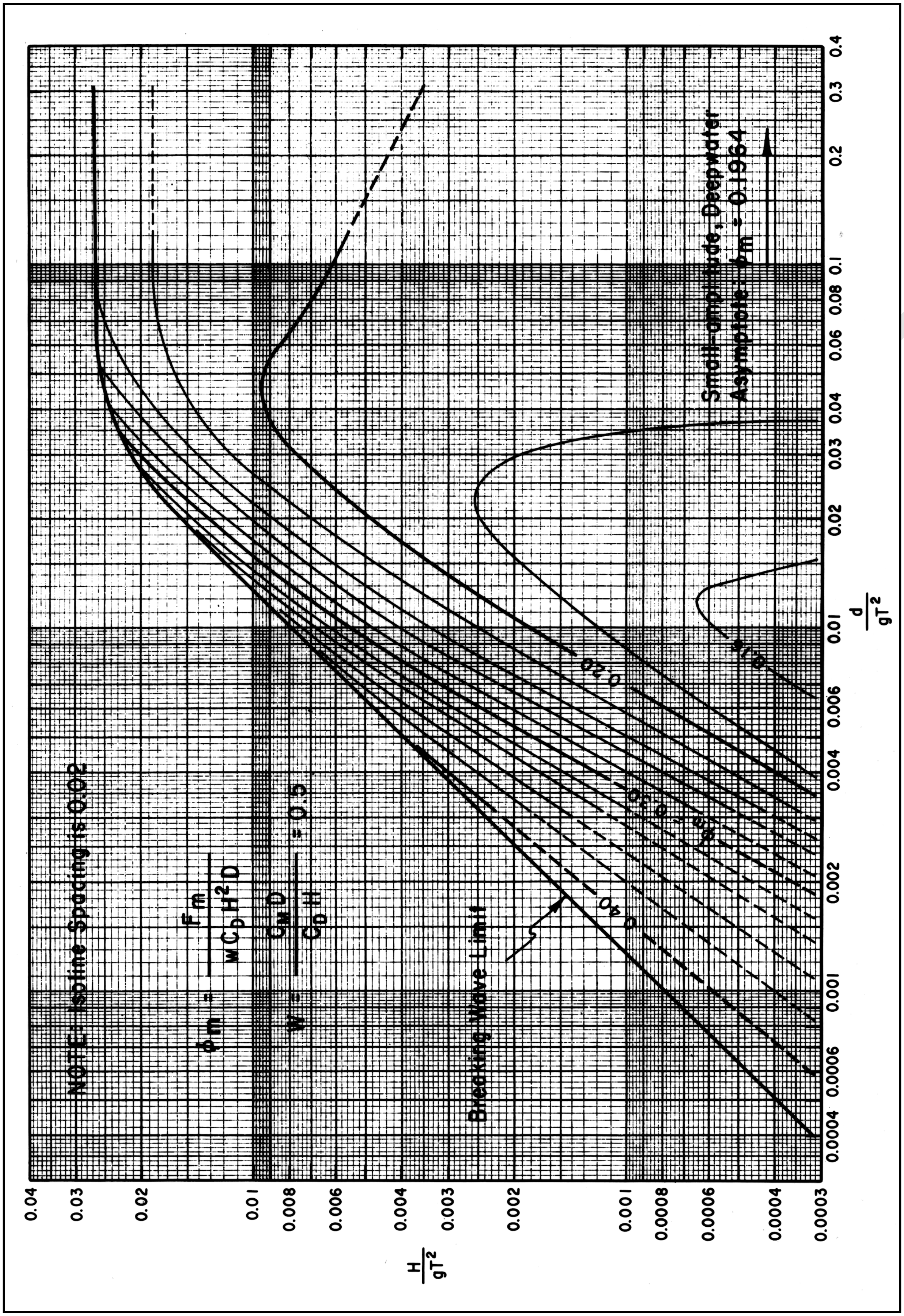

Figure VI-5-133. Isolines of φ_m versus H/gT^2 and d/gT^2 ($W = 0.50$)

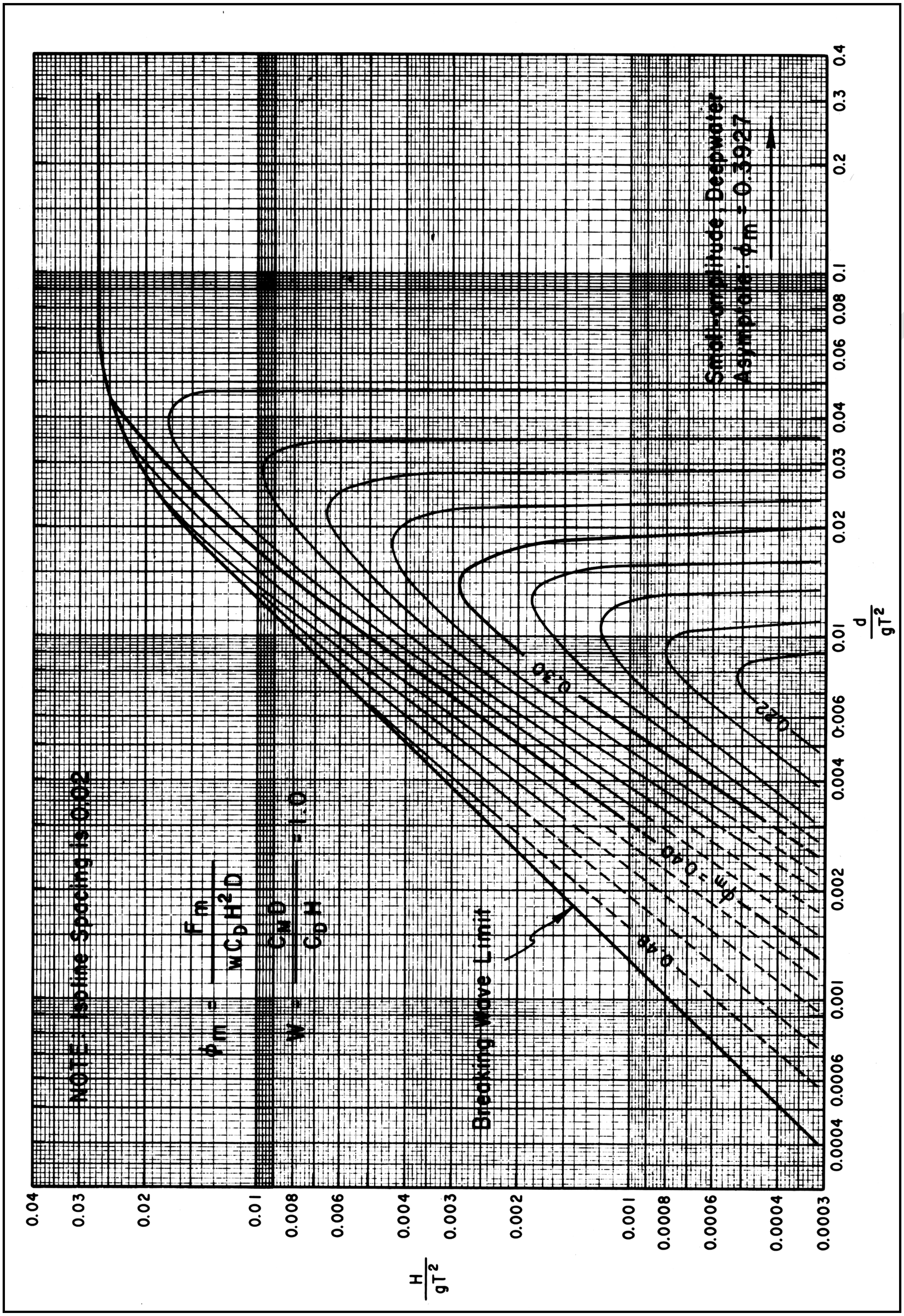

Figure VI-5-134. Isolines of φ_m versus H/gT^2 and d/gT^2 (W = 1.0)

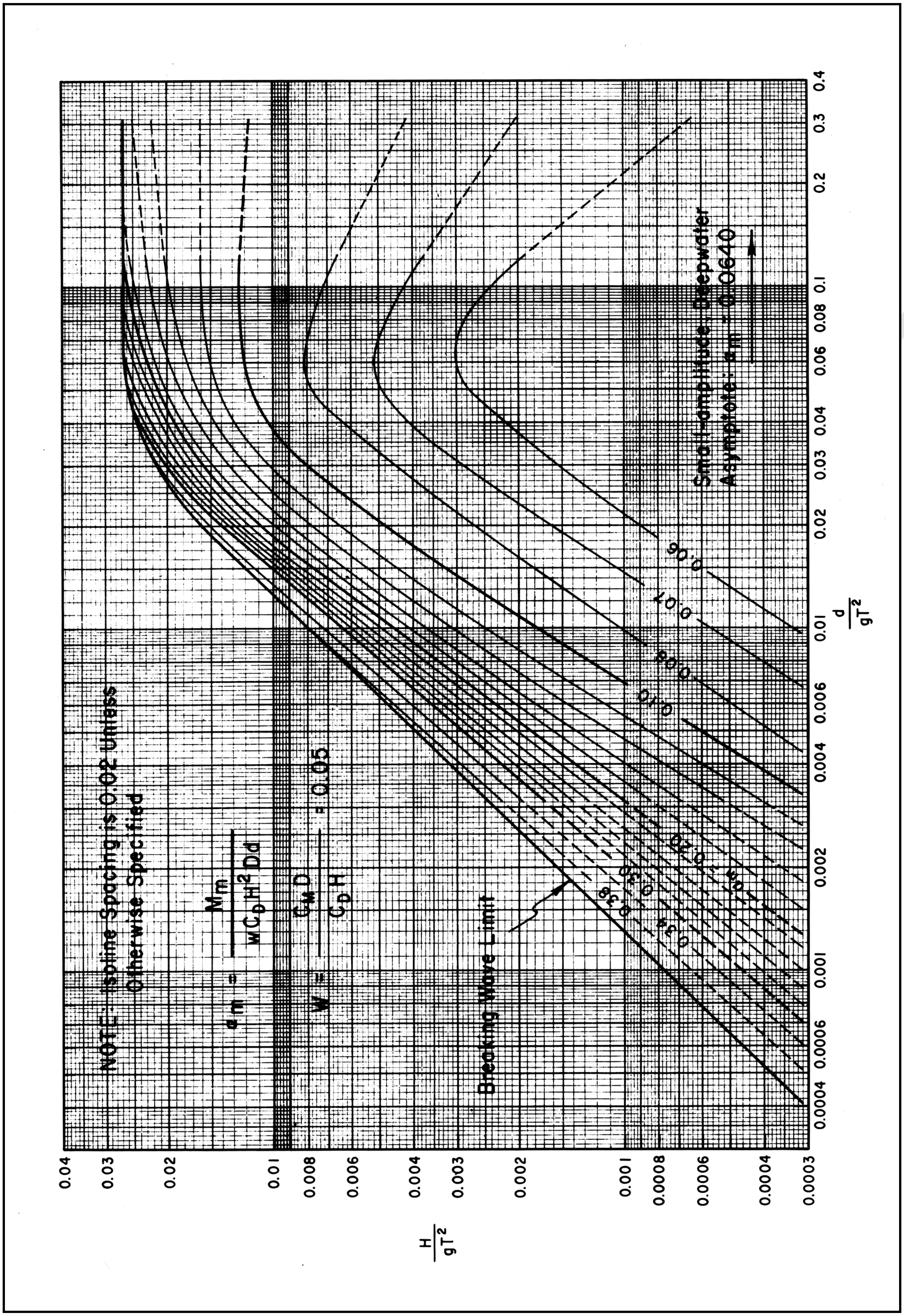

Figure VI-5-135. Isolines of α_m versus H/gT^2 and d/gT^2 (W = 0.05)

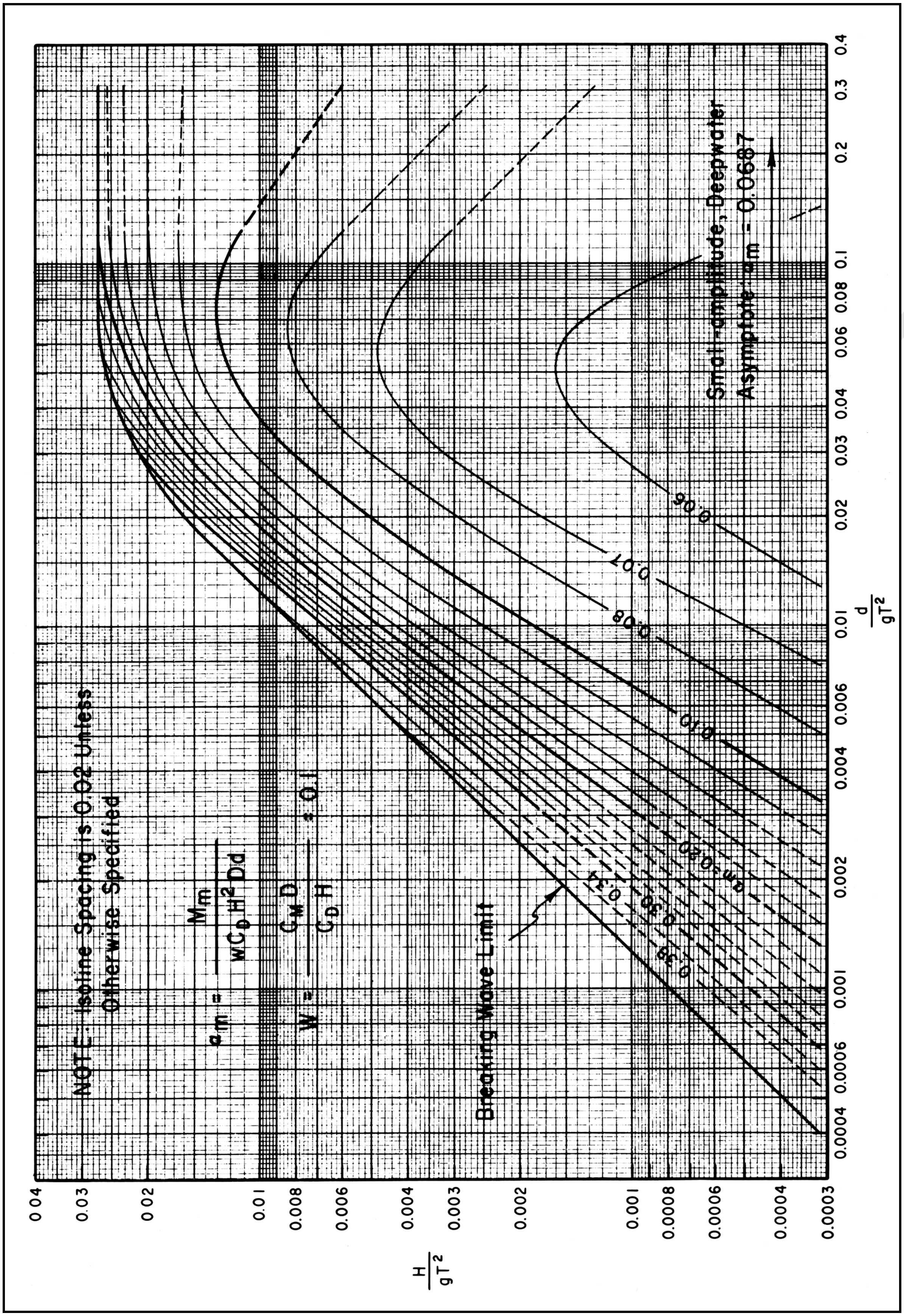

Figure VI-5-136. Isolines of α_m versus H/gT^2 and d/gT^2 (W = 0.10)

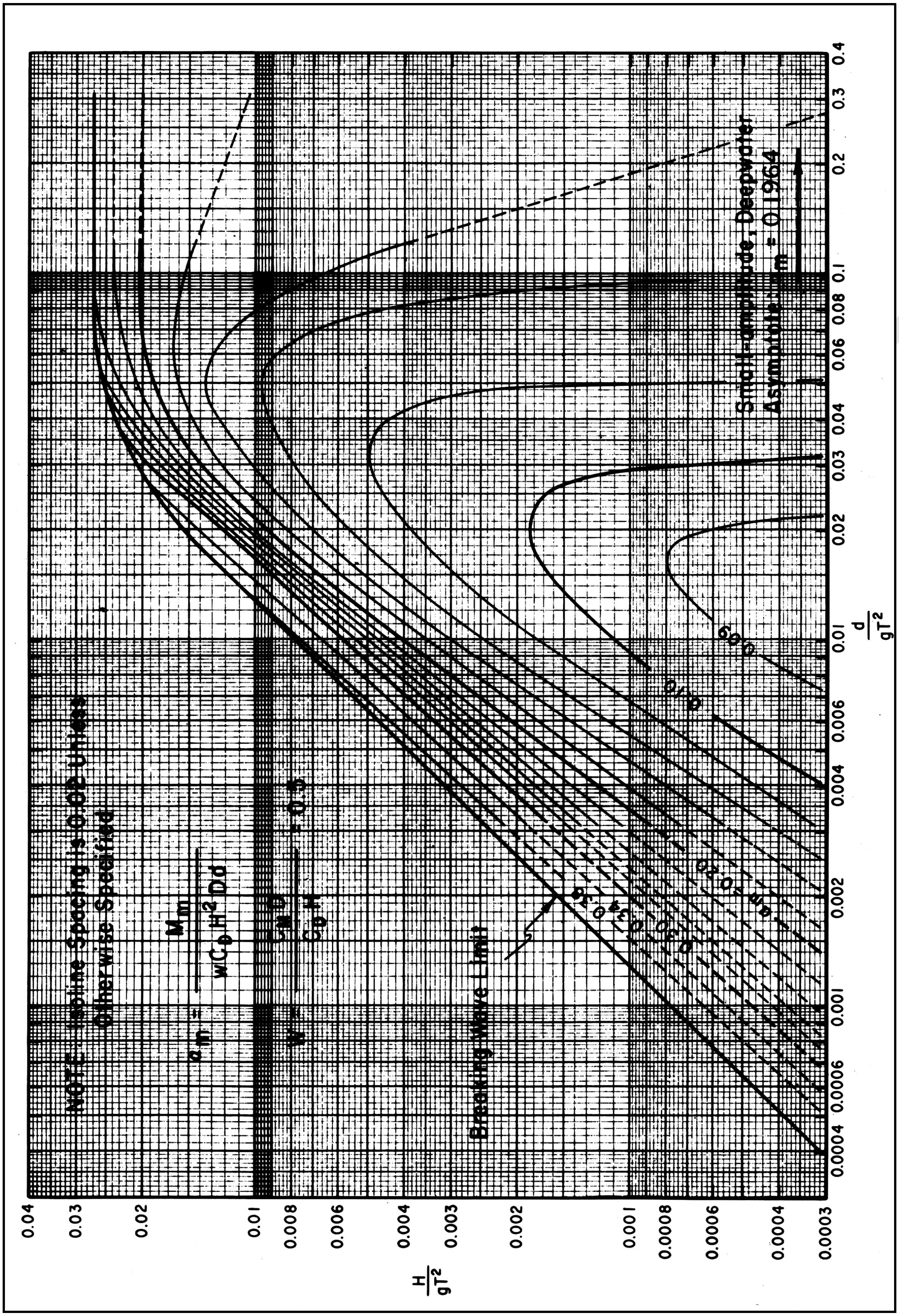

Figure VI-5-137. Isolines of α_m versus H/gT^2 and d/gT^2 ($W = 0.5$)

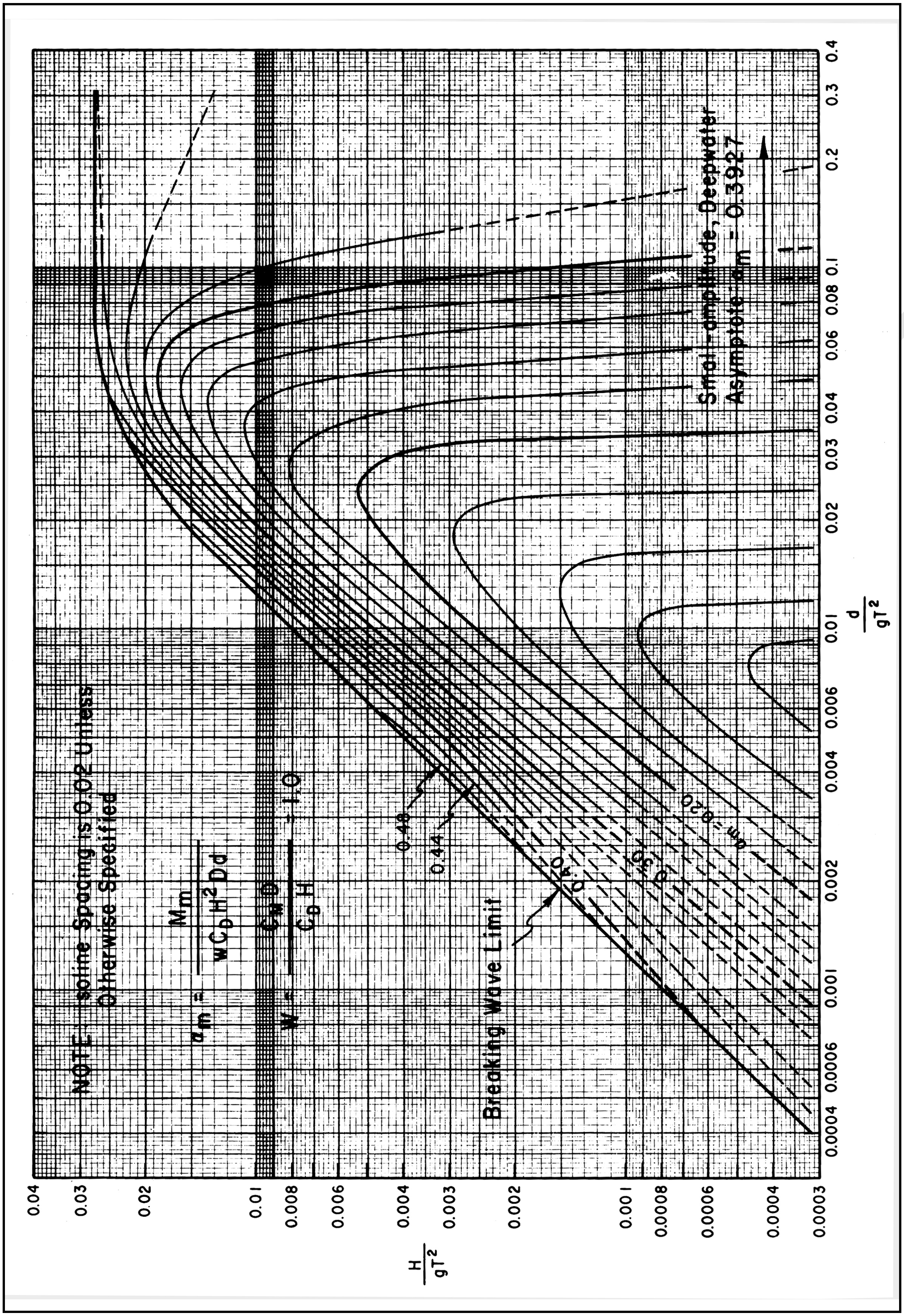

Figure VI-5-138. Isolines of α_m versus H/gT^2 and d/gT^2 ($W = 1.0$)

- For both the total force and total moment calculations, the calculated value of W will likely lie between the values for which the figures are drawn. In this case, determine values of φ_m and α_m from the plots on either side of the W-value, then use linear interpolation to estimate values of φ_m and α_m for the calculated value of W.

- The maximum moment is calculated at the mudline, and the corresponding moment arm is the maximum moment divided by the maximum force, or

$$r_a = \frac{M_m}{F_m} \tag{VI-5-302}$$

- If the surrounding soil does not provide any lateral resistance, or if there has been scour around the pile, the effective moment arm must be increased and a new maximum total moment calculated. For example, if the scour depth beneath the surrounding bed is S_m, the modified maximum total moment will be

$$M_m^{/} = (r_a + S_m)\, F_m \tag{VI-5-303}$$

- See Part VI-7, "Design of Specific Project Elements," for an example illustrating calculation of forces and moments on a vertical cylinder.

(3) Transverse forces due to eddy shedding.

(a) In addition to drag and inertia forces that act in the direction of wave advance, transverse forces may arise. Transverse forces are caused by vortex or eddy shedding on the downstream side of the pile. Eddies are shed alternately from each side of the pile resulting in a laterally oscillating force. Transverse forces act perpendicular to both wave direction and pile axis, and they are often termed lift forces because they are similar to aerodynamic lift acting on an airfoil.

(b) Laird, Johnson, and Walker (1960) and Laird (1962) studied transverse forces on rigid and flexible oscillating cylinders. In general, lift forces were found to depend on the dynamic response of the structure. For structures with a natural frequency of vibration about twice the wave frequency, a dynamic coupling between the structure motion and fluid motion occurs, resulting in large lift forces. Transverse forces have been observed 4.5 times greater than the drag force. However, for rigid structures a transverse force equal to the drag force is a reasonable upper limit. Larger transverse forces can occur where there is dynamic interaction between the waves and cylindrical pile. The design guidance in this section pertains only to rigid piles.

(c) Chang (1964) found in laboratory investigations that eddies are shed at a frequency that is twice the wave frequency. Two eddies are shed after passage of the wave crest (one on each side of the pile), and two are shed on the return flow after passage of the wave trough. The maximum lift force is proportional to the square of the horizontal wave-induced velocity in much the same way as the drag force. Consequently, for design estimates of the lift force the following equation can be applied.

$$F_L = F_{Lm} \cos 2\theta = C_L \frac{\rho g}{2} D H^2 K_{Dm} \cos 2\theta \tag{VI-5-304}$$

where F_L is the time-varying transverse (lift) force, F_{Lm} is the maximum transverse force, θ is the wave phase angle ($\theta = 2\pi x/L - 2\pi t/T$), C_L is an empirical lift coefficient analogous to the drag coefficient in Equation VI-5-286, and K_{Dm} is the dimensionless parameter given in Figure VI-5-127. Chang found that C_L depends on the average Keulegan-Carpenter number given as

$$KC_{ave} = \frac{(u_{\max})_{ave}\ T}{D} \tag{VI-5-305}$$

where $(u_{max})_{ave}$ is the maximum horizontal velocity averaged over the depth. When KC_{ave} is less than 3, no significant eddy shedding occurs and no lift forces are developed. As KC_{ave} increases, C_L increases until it is approximately equal to C_D for rigid piles. Consequently, it must be recognized that: the lift force can represent a major portion of the total force acting on a pile and therefore should not be neglected in the design of the pile.

c. Selection of hydrodynamic force coefficients C_D, C_M, and C_L.

Sarpkaya (1976a, 1976b) conducted an extensive experimental investigation of the inertia, drag, and transverse forces acting on smooth and rough circular cylinders. The experiments were performed in an oscillating U-tube water tunnel for a range of Reynolds numbers up to 700,000 and Keulegan-Carpenter numbers up to 150. Relative roughness of the cylinders k/D varied between 0.002 and 0.02 (where k is the average height of the roughness element). Forces were measured on stationary cylinders, and the corresponding drag and inertia coefficients were determined using a technique of Fourier analysis and least-squares best fit of the Morison equation (Equation VI-5-281) to the measured forces.

The results were presented as plots of the force coefficients versus Keulegan-Carpenter number

$$KC = \frac{u_m\ T}{D} \tag{VI-5-306}$$

for given values of Reynolds number

$$R_e = \frac{u_m\ D}{\nu} \tag{VI-5-307}$$

or the frequency parameter

$$\beta = \frac{R_e}{KC} = \frac{D^2}{\nu T} \tag{VI-5-308}$$

In Equations VI-5-306 - VI-5-308 u_m is the maximum horizontal wave velocity, T is the wave period, D is the cylinder diameter, and ν is the fluid kinematic viscosity.

Figures VI-5-139 through VI-5-141 present Sarpkaya's (1976a, 1976b) experimental results for the force coefficients C_D, C_M, and C_L for smooth cylinders. In each figure the force coefficient is plotted versus Keulegan-Carpenter number for constant values of Reynolds number (dotted lines) and frequency parameter (solid lines). Drag and inertia force coefficients versus Reynolds number for rough cylinders are plotted on Figures VI-5-142 and VI-5-143, respectively, for selected values of relative roughness k/D. Sarpkaya cautioned that the force coefficients were developed for oscillatory flow with zero mean velocity, and it is possible that waves propagating on a uniform current may have different force coefficients.

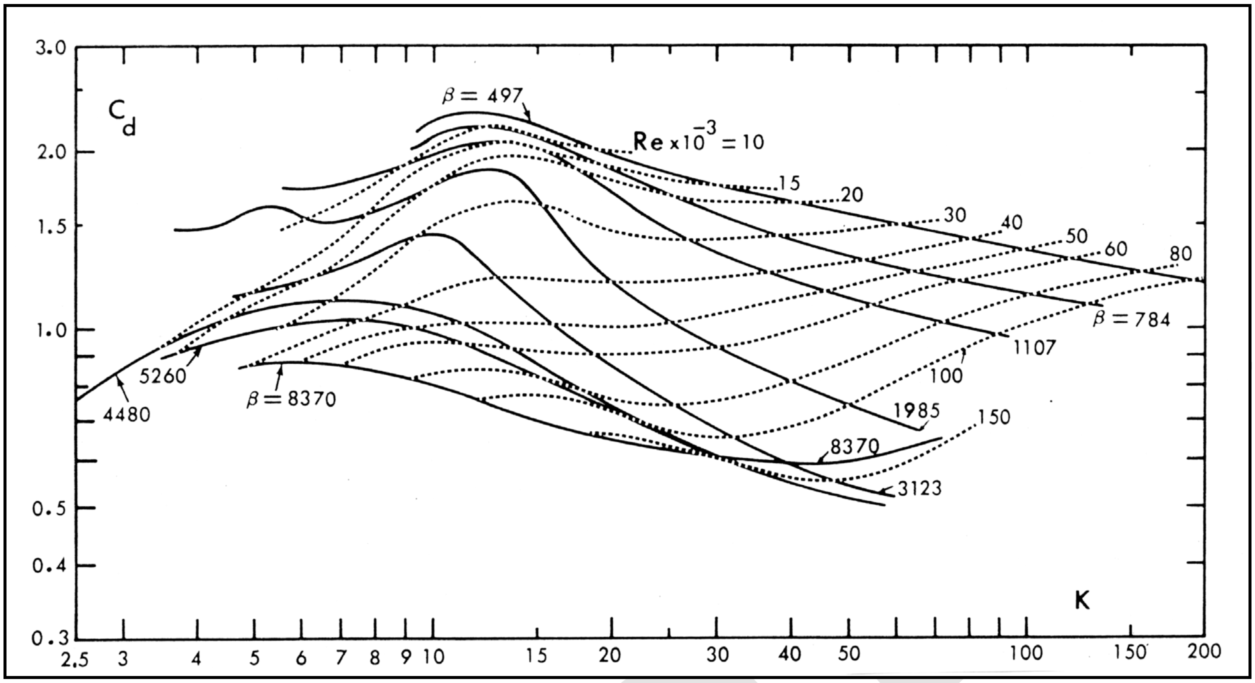

Figure VI-5-139. Drag coefficient C_D as a function of *KC* and constant values of R_c or β for smooth cylinders (from Sarpkaya 1976a)

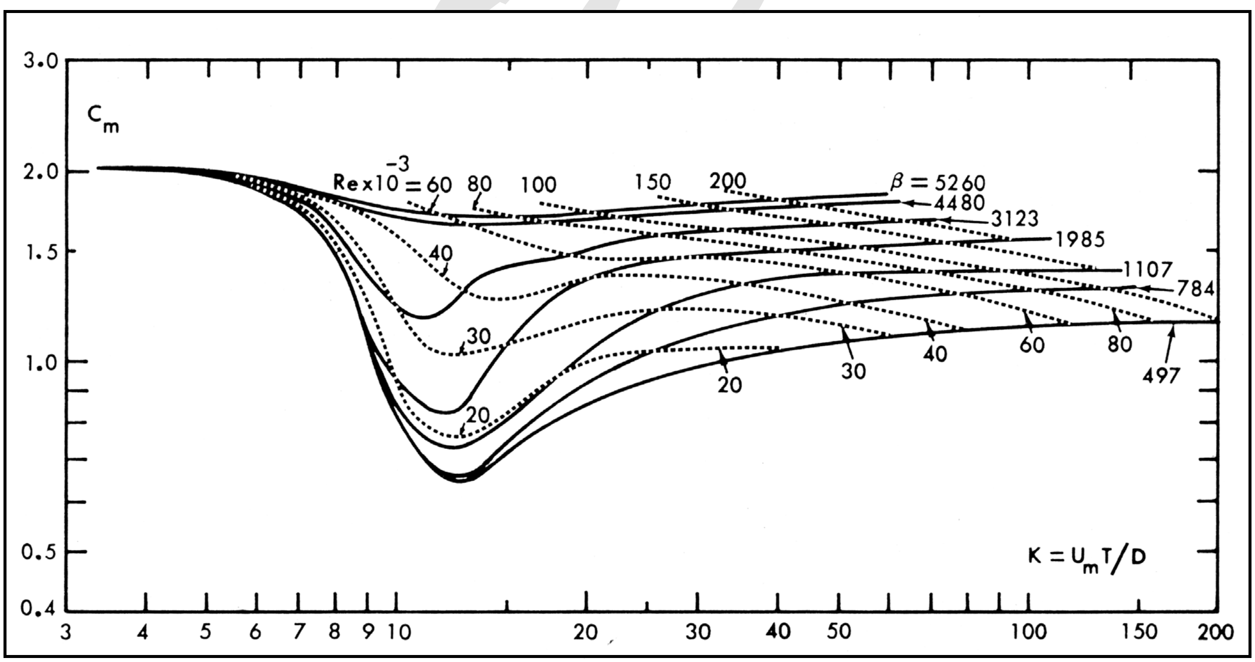

Figure VI-5-140. Inertia coefficient C_M as a function of KC and constant values of R_e or β for smooth cylinders (from Sarpkaya 1976a)

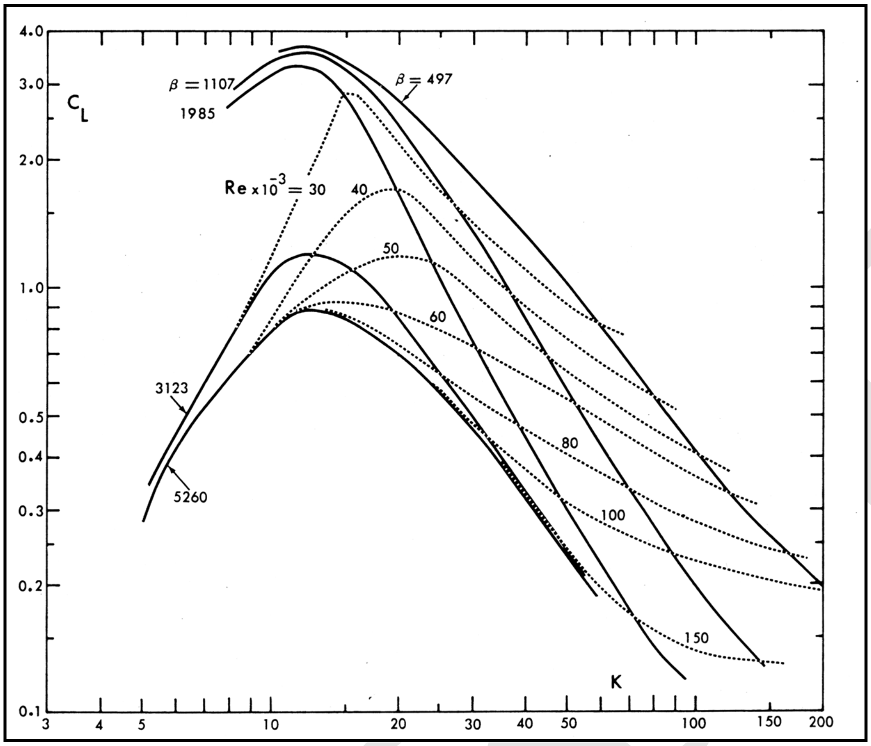

Figure VI-5-141. Lift coefficient C_L as a function of *KC* and constant values of R_e or β for smooth cylinders (from Sarpkaya 1976a)

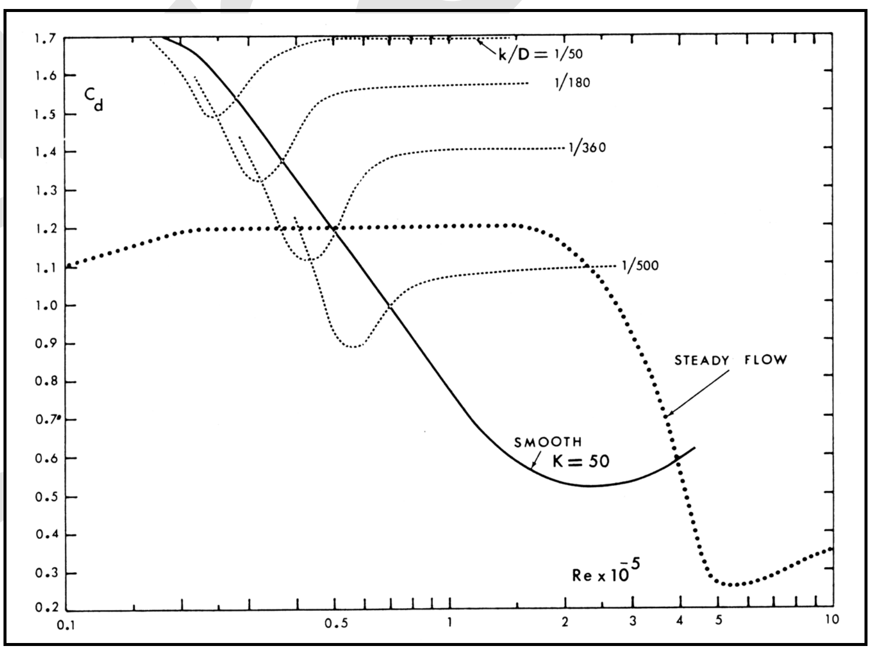

Figure VI-5-142. Drag coefficient C_D as a function of Reynolds number for rough cylinders (from Sarpkaya 1976a)

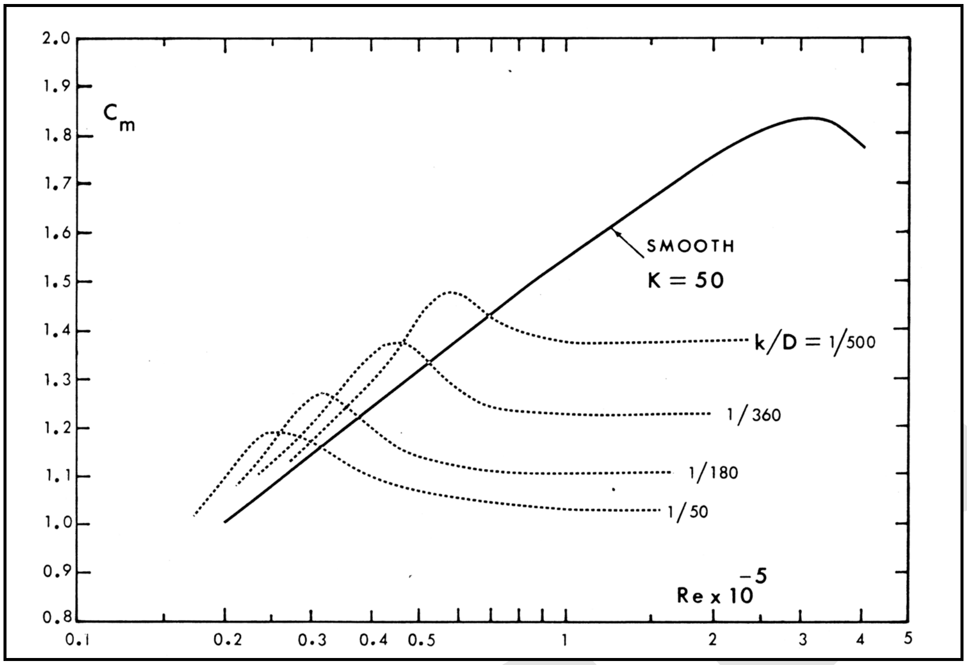

Figure VI-5-143. Inertia coefficient C_M as a function of Reynolds number for rough cylinders (from Sarpkaya 1976a)

The force coefficients given in Figures VI-5-139 through VI-5-143 should give reasonable force estimates when used with the design figures based on stream function theory given in the previous section. However, the design engineer should be aware of the limitations of assuming the force coefficients are constant over the water depth and throughout the wave cycle.

Sarpkaya's experimental apparatus gave uniform values of Reynolds number and Keulegan-Carpenter number over the entire test pile. For a vertical pile exposed to waves, the maximum horizontal velocity will vary from its largest value at the sea surface to a somewhat smaller value near the bottom. Consequently, both R_e and KC will vary over the depth of the pile. For design purposes, it is reasonable to calculate R_e and KC based on the average value of u_m over the water depth in shallow water because the variation will not be too significant. In deeper water it may be wise to investigate the variation of force coefficients with depth to determine if using R_e and KC based on average u_m is appropriate.

Sarpkaya's experimental data do not cover the range of Reynolds numbers likely to be encountered with bigger waves and larger pile diameters. For larger calculated Reynolds numbers use the following guidance that has been repeated from the old Shore Protection Manual.

$$C_D = \begin{cases} 1.2 - \dfrac{(R_e - 2(10)^5}{6(10)^5} & \text{for} \quad 2(10)^5 < R_e - 5(10)^5 \\ 0.7 & \text{for} \quad 5(10)^5 < R_e \end{cases}$$

$$C_M = \begin{cases} 2.5 - \dfrac{R_e}{5(10)^5} & \text{for} \quad 2.52(10)^5 < R_e - 5(10)^5 \\ 1.5 & \text{for} \quad 5(10)^5 < R_e \end{cases}$$

Bear in mind the above recommendations for higher Reynolds number are based on older experimental results, and more accurate estimates might be available from the offshore engineering literature for critical applications.

d. Safety factors in pile design.

Before the pile is designed or the foundation analysis is performed, a safety factor is usually applied to calculated forces. Reasons for uncertainty to the design include approximations in applying the wave theory, estimated values for the force coefficients, potential loss of pile strength over time, and the probability that the design wave will be exceeded during the life of the structure.

The following recommendations for safety factors are offered as general rules of thumb. In situations where pile failure could lead to loss of life or catastrophic failure of supported infrastructure, safety factors should be increased.

(a) When the design wave has low probability of occurrence, it is recommended that a safety factor of 1.5 be applied to calculated forces and moments that are to be used as the basis for structural and foundation design.

(b) If the design wave is expected to occur frequently, such as in depth-limited situations, a safety factor of at least 2.0 should be applied to the calculated forces and moments.

In addition to the safety factor, changes occurring during the expected life of the pile should be considered in design. Such changes as scour about the pile base and added pile roughness due to marine growth may be important.

The design procedure presented in the previous sections is a static procedure; forces are calculated and applied to the structure statically. The dynamic nature of forces from wave action must be considered in the design of some offshore structures. When a structure's natural frequency of oscillation is such that a significant amount of energy in the wave spectrum is available at that frequency, the dynamics of the structure must be considered. In addition, stress reversals in structural members subjected to wave forces may cause failure by fatigue. If fatigue problems are anticipated, the safety factor should be increased or allowable stresses should be decreased. Evaluation of these considerations is beyond the scope of this manual.

Corrosion and fouling of piles also require consideration in design. Corrosion decreases the strength of structural members. Consequently, corrosion rates over the useful life of an offshore structure must be estimated and the size of structural members increased accordingly.

Fouling of a structural member by marine growth increases the roughness and effective diameter of the pile and also changes the values of the force coefficients. The increased diameter must be carried through the entire design procedure to determine forces on the fouled member.

e. Other considerations related to forces on slender cylindrical piles.

(1) Wave forces on pile groups. For a group of piles supporting a structure such as a platform or pier, the methods given in the previous sections can be used provided the piles are sufficiently separated so that flow around one pile does not influence the flow around adjacent piles. One approach is to assume waves are long crested and of permanent form. Given the relative orientation of the piles to each other and to the incoming wave, forces can be estimated on each pile at different times during the wave passage. Typically, the maximum force on individual piles occurs at different times unless all the piles are parallel to the wave crest. Therefore, numerous calculations throughout the wave passage are needed to determine the worst

loading on the overall structure. Because the tops of the piles are connected by the superstructure, and the connections may provide some rigidity; it may be necessary to analyze the pile group as a frame.

As the distance between piles becomes small relative to the wavelength, maximum forces and moments on pile groups may be conservatively estimated by summing the maximum forces and moments on each pile.

The assumption that piles are unaffected by neighboring piles is not valid when the distance between piles is less than about three times the pile diameter. Chakrabarti (1991) presented design graphs giving maximum force on a pile in a linear pile group (piles aligned in a row) as a function of Keulegan-Carpenter number and relative separation distance S/D where S is the distance between center lines of adjacent piles. Graphs were provided for pile groups consisting of two, three and five piles with waves approaching parallel and perpendicular to the line of piles. Graphs were also given for estimating C_D, C_M, and C_L for pile groups of three and five piles exposed to waves parallel and perpendicular to the pile line.

(2) Wave forces on nonvertical piles. Forces and moments on nonvertical cylindrical piles can be estimated using Morison's equation (Equation VI-5-281) where the values for velocity u and acceleration du/dt are given as the velocity and acceleration components perpendicular to the pile. Calculations will need to be performed using an appropriate wave theory along with the force coefficients given in Part VI-5-7-c, "Selection of hydrodynamic force coefficients C_D, C_M, *and* C_L." Do not use the curves provided in design Figures VI-5-126 through VI-5-129 and VI-5-131 through VI-5-134 because these figures are only for vertical piles. For nonvertical piles, the pile self weight (immersed and above water) will contribute to the overturning moment and must be included in the calculation.

(3) Broken wave forces. Forces resulting from action of broken waves on piles are much smaller than forces due to breaking waves. When pile-supported structures are constructed in the surf zone, lateral forces from the largest wave breaking on the pile should be used for design. Although breaking-wave forces in the surf zone are great per unit length of pile, the pile length actually subjected to wave action is usually short. Hence, the resulting total force and moment are small. Pile design in the surf zone is usually governed primarily by vertical loads acting along the pile axis.

VI-5-8. Other Forces and Interactions

a. Impact forces. Impact force loading on coastal projects occurs when waves or solid objects collide with typically stationary coastal structure elements. Only solid body impacts are discussed in this section. Impact loads between shifting concrete armor units are discussed in Part VI-5-3-c, "Structural integrity of concrete armor units."

Certain coastal structures such as thin-walled flood barriers, sheet-pile bulkheads, mooring facilities, coastal buildings, or other infrastructure may be subject to impact damage by solid objects carried by waves, currents, or hurricane-force winds. During severe storms, high winds may propel small pleasure craft, barges, and floating debris which can cause significant horizontal impact loads on structures. Likewise, floating ice masses can also cause great impact loads. Impact loads are an important consideration in design of vessel moorings and fendering systems.

Designing a structure to resist impact loads during extreme events is difficult because of uncertainty associated with impact speed and duration. In situations where impact damage by large floating objects could cause catastrophic loss, it may be prudent to limit adjacent water depth by constructing sloping rubble-mound protection fronting the structure or by placing submerged breakwaters seaward of the structure to ground large floating masses and eliminate the hazard.

Impact forces are evaluated using impulse-momentum and energy considerations found in textbooks on fundamental dynamics. However, application of these principles to particular impact problems is difficult unless reliable estimates can be made of object mass (including added mass in water), the mass initial and final velocities, duration of impact loading, and distribution of impact force over time. In addition, some evaluation must be made on whether the collision of the floating object with a coastal structure results in purely elastic behavior in which momentum is conserved, purely plastic impact with all the kinetic energy of the impact being absorbed, or some combination of the two.

Fendering systems in ports and harbors are designed to absorb low-velocity impacts by vessels during docking maneuvers and seiching motions. Design of fendering systems is adequately covered by numerous textbooks and design standards. Examples of typical design references in the coastal engineering literature include Quinn (1972) and Costa (1973). The modes of kinetic energy absorption by fendering systems were studied theoretically by Hayashi and Shirai (1963). Otani, Horii, and Ueda (1975) presented field measurements related to absorption of impact kinetic energy of 50 large tankers. They observed that most berthing velocities are generally below 6 cm/s, and that measured impact energy was substantially larger than calculated using the design standards that existed at that time. Kuzmanovic and Sanchez (1992) discussed protective systems for bridge piers and pilings, and they gave procedures for accessing the equivalent static force acting on bridge piers due to vessel impacts.

b. Ice forces. A description of ice loading and how it may impact various types of coastal structures in the context of site-specific design criteria is given in Part VI-3-5, "Ice." Other general references include Chen and Leidersdorf (1988); Gerwick (1990); and Leidersdorf, Gadd, and Vaudrey(1996). The following section presents methods for calculating ice forces under specific loading conditions.

(1) Horizontal ice forces.

(a) Solid ice forces.

- Large horizontal forces can result when solid sheet ice, or large chunks of solid ice that have broken free, come in contact with vertical-front coastal structures. Most ice sheets are large enough that impact forces are limited by ice failure in the weakest mode permitted by the mechanics of interaction as the structure penetrates the ice, i.e., crushing, splitting, shear, or bending. For smaller ice blocks or wide structures, the maximum impact force may be limited by the kinetic energy available at the moment of impact (HQUSACE 1982). Ice impact calculations should be based on impulse-momentum considerations, but such calculations will be difficult because of uncertainty in estimating a value for ice block velocity.

- Wind and water current drag acting on large floating blocks of ice press the ice blocks against structures creating large pressures at the points of contact. The force due to drag on a block of ice can be calculated for wind and water currents using the following formula (PIANC 1992)

$$F_d = C_{sf}\, \rho\, A\, (u - u_i)^2 \qquad \text{(VI-5-309)}$$

where

C_{sf} = coefficient of skin friction between wind and ice or water and ice (see Table VI-5-89)

ρ = fluid density (air or water)

Table VI-5-89
Values of Skin Friction Coefficient, C_{sf} (PIANC 1992)

	Smooth Ice	Rough (Pack) Ice
Wind drag	0.001 - 0.002	0.002 - 0.003
Water drag	0.002 - 0.004	0.005 - 0.008

A = horizontal area of ice sheet

u = fluid velocity (10 m above ice for air or 1 m below ice for water)

u_i = velocity of ice in the direction of u

- Separate drag calculations should be performed for both wind and water currents with the results treated as vector forces on the ice mass. Because drag force is directly proportional to ice surface area, larger ice sheets will exert greater forces.

- Once an ice sheet has come to rest against a structure, u_i is zero, and the total drag force can be calculated. Intact ice sheets should be treated as solid bodies with the resultant loads vectorially distributed among the structure/ice contact points using force and moment balance. The total force may be somewhat uniformly distributed along a lineal vertical wall. However, if the ice block comes in contact at only a few discrete points, the contact pressure may be very large. In these cases, the calculated force due to drag may exceed the force necessary for local crushing of the ice, in which case the local crushing strength becomes the limiting force applied to the structure.

(b) Localized ice crushing forces.

- The limiting ice force on a vertical structure is determined by the crushing failure strength of the ice in compression. A theoretical expression for the horizontal ice crushing force was given in Korzhavin in a 1962 Russian publication (Ashton 1986) as

$$\frac{F_c}{bh_i} = m\ I\ k\ x\ \sigma_c \tag{VI-5-310}$$

where

F_c = horizontal crushing force

b = structure horizontal width or diameter

h_i = thickness of ice sheet

m = plan shape coefficient

I = indentation coefficient

k = contact coefficient

x = strain rate function

σ_c = ice compressive failure strength in crushing

- This formula is usually applied to piles and pier structures rather than long vertical walls. The plan shape coefficient, m, is 1.0 for flat surfaces, 0.9 for circular piles, and $0.85[\sin(\beta/2)]^{1/2}$ for wedge-shaped structures having a wedge angle of β. The indentation coefficient, I, has been found experimentally to be a function of the aspect ratio, b/h_i, and it is usually presented in graphical form. The contact coefficient, k, is a function of ice velocity and width of structure, and it varies between values of 0.4 to 0.7 for ice velocities between 0.5 and 2.0 m/s. The strain rate coefficient is also a function of ice speed. Ashton (1986) provided further details about the theoretical development of Equation VI-5-310 and its associated coefficients.

- In a Russian publication, Afansev (Ashton 1986) combined the coefficients I, k, and x of Equation VI-5-310 into a single coefficient, C, giving the formula

$$\frac{F_c}{bh_i} = C\ m\ \sigma_c \tag{VI-5-311}$$

- Afansev established the following empirical relationship for C based on model tests using laboratory-grown, saline ice.

$$\begin{aligned} C &= \left(5\ \frac{h_i}{b} + 1\right)^{1/2} && \text{for} \quad 1 < \frac{b}{h_i} \\ C &= 4.17 - 1.72\left(\frac{b}{h_i}\right) && \text{for} \quad 0.1 < \frac{b}{h_i} < 1 \end{aligned} \tag{VI-5-312}$$

- The lower formula in Equation VI-5-312 is a linear interpolation as recommended in Ashton (1986).

- In Equation VI-5-311 values of the shape coefficient are the same as given for the Korzhavin formula (Equation VI-5-310).

- The Canadian Standards Association Bridge Code (Canadian Standards Association 1978) recommended an even more simplified version of Equation VI-5-310 given by

$$\frac{F_c}{bh_i} = \sigma_c \tag{VI-5-313}$$

using the range of values for sheet ice compressive crushing strength shown in Table VI-5-90. Equation VI-5-313 and the crushing strength values of Table VI-5-90 were also adopted by the American Association of State Highway and Transportation Officials.

Table VI-5-90
Values of Effective Ice Crushing Strength, σ_c

Ice Crushing Stress	Environmental Situation
0.7 MPa (100 psi)	Ice breakup occurs at melting temperatures and the ice moves in small pieces that are essentially disintegrated.
1.4 MPa (200 psi)	Ice breakup occurs at melting temperatures, but the ice moves in large pieces that are generally sound.
2.1 MPa (300 psi)	Ice breakup consists of an initial movement of the entire ice sheet or large sheets of sound ice impact piers.
2.8 MPa (400 psi)	Ice breakup occurs with an ice temperature significantly below the melting point and ice movement consists of large sheets.

- Use of Equation VI-5-313 implies that the product $C \cdot m = 1$ in Equation VI-5-311, which corresponds to large values of b/h_i. This is a realistic assumption for large bridge piles and piers, but ice crushing forces on smaller diameter piles should be calculating using the appropriate strength values from Table VI-5-90 in Equation VI-5-311.

(c) Thermal ice forces. Equations are available for predicting ice temperature based on an energy balance between the atmosphere and the ice sheet. However, the required parameters (air temperature, air vapor pressure, wind, and cloud cover) needed to calculate thermal expansion are difficult to estimate. Thermal strain is equal to the ice thermal expansion coefficient times the change in ice temperature. For restrained or partially restrained ice sheets a nonlinear, time dependent stress-strain law is used to predict thermal stresses (HQUSACE 1982). Because of stress relaxation due to creep, the rate of thermal change is an important factor; and even a thin snow cover can drastically reduce thermal stresses in ice sheets.

- A design rule-of-thumb for thermal expansion loads per unit horizontal length on dams and other rigid structures is 145 - 220 kN/m (10,000 - 15,000 lbs/ft) (HQUSACE 1982). Movable structures should allow for 73 kN/m (5,000 lbs/ft). These values are based on field measurements.

- Thermal expansion of water frozen between elements of a coastal structure can result in dislocation of individual elements or cracking of armor units making the protection vulnerable to wave attack.

(2) Ice forces on slopes.

(a) Ride-up of ice on slopes.

- When horizontally moving ice encounters a sloping structure, a component of the horizontal force pushes the ice up the slope. This action induces a bending failure in the ice sheet at loads less than required for ice crushing failure. Ashton (1986) showed the derivation of a simple two-dimensional theory for calculating the horizontal force exerted by ice on a sloping structure as illustrated in Figure VI-5-144. (Ashton also included discussion and analysis of the more complex case of ice ride-up on three-dimensional structures).

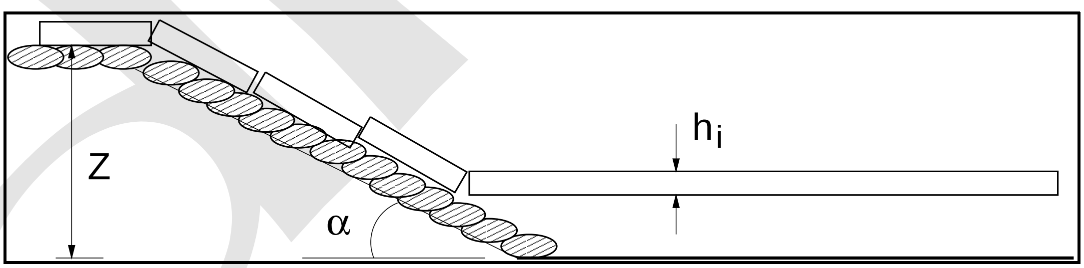

Figure VI-5-144. Ice riding up on structure slope

- For the two-dimensional case the horizontal force per unit width of structure was given by the expression

$$\frac{F_h}{b} = C_1 \, \sigma_f \left(\frac{\rho_w \, g h_i^5}{E} \right)^{1/4} + C_2 \, Z \, \rho_i \, g h_i \qquad \text{(VI-5-314)}$$

with

$$C_1 = 0.68 \left[\frac{\sin \alpha + \mu \cos\alpha}{\cos \alpha - \mu \sin \alpha} \right] \qquad \text{(VI-5-315)}$$

and

F_h = total horizontal force

$$C_2 = \left[\frac{(\sin \alpha + \mu \cos \alpha)^2}{\cos \alpha - \mu \sin \alpha} + \frac{\sin \alpha + \mu \cos \alpha}{\tan \alpha} \right] \qquad \text{(VI-5-316)}$$

b = horizontal width of structure contact zone

h_i = ice sheet thickness

σ_f = flexural strength of ice (0.5 = 1.5 MPa)

ρ_w = water density

ρ_i = ice density (915 = 920 kg/m^3)

E = modulus of elasticity of ice (1,000 = 6,000 MPa)

Z = maximum vertical ice ride-up distance

g = gravitational acceleration

α = structure slope angle relative to horizontal

μ = structure slope friction factor (0.1 = 0.5)

- The first term in Equation VI-5-314 is interpreted as the force necessary to break the ice in bending, and the second term is the force that pushes the ice blocks up the sloping structure. The modulus of elasticity varies from 1,000 MPa for very salty water up to about 6,000 MPa for fresh water (Machemehl 1990). Ashton (1986) warned that this simple two-dimensional theory will be inadequate for narrow structures because the zone of ice failure will be wider than the structure.

- Low values of friction factor ($\mu = 0.1$) are associated with smooth slopes such as concrete or carefully layed block protection, whereas high values (μ = 0.5) are applicable for randomly-placed stone armor, riprap, or filled geotextile bags. For slopes steeper than 1:1, the horizontal ice force increases rapidly for the higher friction factors, and there is a risk of the dominant failure mode being crushing or buckling rather than bending. Milder slopes with smooth surfaces are much more effective in reducing horizontal ice forces. Croasdale, Allyn, and Roggensack (1988) discussed several additional aspects related to ice ride-up on sloping structures.

- Quick "rough" estimates of horizontal forces on sloping structures can be made using a variation of Equation VI-5-313 as proposed in Ashton (1986), i.e.,

$$\frac{F_h}{b} = K_h \; h_i \; \sigma_c \qquad \text{(VI-5-317)}$$

where K_h is approximated from a curve given in Ashton (1986) by the formula

$$K_h = 1 - 0.654 \, f^{0.38} \qquad \text{(VI-5-318)}$$

with

$$f = \frac{1 - \mu \tan \alpha}{\mu + \tan \alpha} \qquad \text{(VI-5-319)}$$

and σ_c is the ice compressive strength as given in Table VI-5-90. As slope angle increases, K_h approaches a value of unity which represents failure by crushing. For decreasing slope angles, K_h decreases because of the increasing tendency of the ice to fail in bending. Values of K_h less than 0.2 should never to used in Equation VI-5-317.

(b) Adfreeze loads. When ice that is in contact with a coastal structure is stationary for a sufficient time, the ice will freeze to the structure or its elements. Adfreeze loads result if the ice then moves either horizontally by dynamic ice thrust or vertically due to changing water level. This is more of a problem in lakes with slowly varying water levels than in tidal waters.

- Little guidance is available on adfreeze stresses with adhesion strength varying between 140 kPa to 1050 kPa for freshwater ice (PIANC 1992). Adfreeze may dislodge individual armor stones on rubble-mound slopes creating a weakness in the armor layer. This can be prevented by using oversized stones or interlocking armor on the slope. A survey of riprap structures at Canadian hydropower reserviors concluded that plucking of individual stones frozen to ice could be largely prevented by sizing the riprap median diameter (d_{50}) greater than the expected maximum winter ice thickness (Wuebben 1995).

(3) Vertical ice forces. Ice frozen to coastal structures can create vertical forces due to ice buoyancy effects when water level rises, or by ice weight when water level falls. These vertical forces will persist until the ice sheet fractures due to bending or the adfreeze force is exceeded.

(a) Cylindrical piles.

- In cases where the ice sheet freezes around a pile, forces will be exerted on the pile if the water level rises or falls. A rising water level will lift the ice sheet, and under certain conditions the uplift force on the pile may be sufficient to pull the pile free. Similarly, during falling water levels the weight of the ice sheet will exert a downward force on a pile which may be sufficient to buckle a slender pile.

- Kerr (1975) studied vertical loads on cylindrical piles and presented equations for calculating loads under the conservative assumption that the water level change is rapid enough to assure elastic ice behavior before failure. A closed-form solution to the governing equation was obtained in terms of Bessel functions, and Kerr presented a numerically evaluated solution in graphical form as shown on Figure VI-5-145. The graphical solution is dimensional, and it has the functional form of

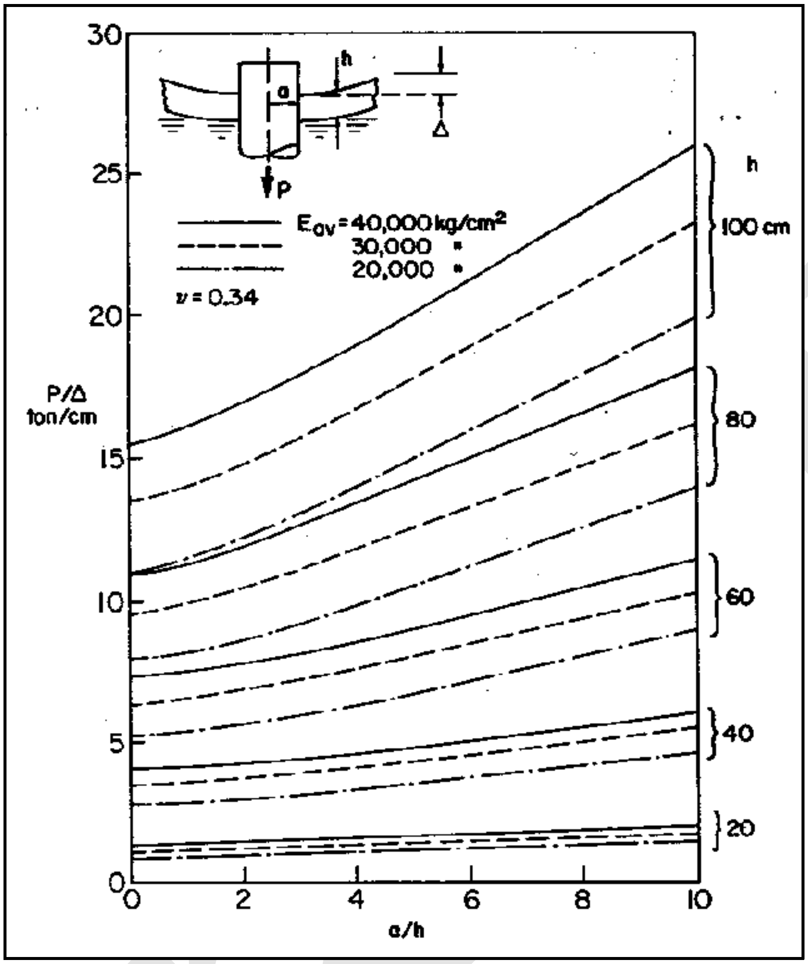

Figure VI-5-145. Vertical ice forces on a cylindrical pile (Kerr 1975)

$$P = f(a,\ h,\ E_{av},\ \Delta,\ \nu) \qquad \text{(VI-5-320)}$$

where

P = uplift force in metric tons (tonnes)

a = pile radius (cm)

h = ice plate thickness (cm)

E_{av} = averaged Young's modulus for ice (kg/cm^2)

Δ = water level rise (cm) up to the thickness of the ice

ν = Poisson's ratio

- Kerr's solution gives estimates of the maximum vertical load assuming the ice sheet does not fail in shear or bending before the maximum load on the pile is reached. For example, the maximum uplift force on a pile with radius a = 100 cm surrounded by a 40-cm-thick ice sheet having an average

Young's modulus of 30,000 kg/cm^2 would be estimated from Figure VI-5-145 using a value of *a/h* = 2.5 giving $P_{max} = 3.7\,\Delta$. The total maximum force for a 5-cm water level rise would be

$$P_{\max} = 3.7\ \Delta = 3.7\ (5\ cm) = 18.5\ tonnes$$

- Kerr (1975) pointed out that the same analysis applied for falling water levels with only a sign change, thus Figure VI-5-145 can also be used decreasing water levels.

(b) Vertical walls.

- Uplift or downward forces per unit horizontal length caused by vertical movement of ice sheets frozen to vertical walls can be approximated using the following formula (PIANC 1992)

$$\frac{F_v}{b} = \rho_w g\ \Delta h\ L_c \qquad \text{(VI-5-321)}$$

where the characteristic length L_c is given as

$$L_c = \left[\frac{E h_i^3}{12\rho_w g(1 - \nu^2)}\right]^{1/4} \qquad \text{(VI-5-322)}$$

and

F_v = total vertical force acting on the wall

b = horizontal length of wall

Δh = change in water level

ρ_w = density of water

g = gravitational acceleration

E = modulus of elasticity of ice

h_i = ice thickness

ν = Poisson's ratio (0.31-0.35)

- As previously mentioned, the modulus of elasticity for ice varies with brine volume from about 1,000 MPa for very salty water to about 6,000 MPa for freshwater ice. For freshwater ice, L_c is typically between 15 to 20 times the ice thickness with a reasonable rule-of-thumb being $L_c \approx 17\ h_i$.

(c) Sloping structures. The additional vertical load caused by the ride-up and piling of ice on sloping structures needs to be evaluated for the local conditions and specific type of structure. Ice piled up on the slope could initiate slumping of the armor layer on steeper slopes. During rising waters, individual armor stones or revetment units might be lifted out by adfreeze forces.

(4) Aspects of slope protection design.

(a) Much of our understanding of successful slope protection design in cold coastal regions stems from practical experience as documented in the technical literature. In general the design philosophy recognizes that little can be done to prevent ice contact with slope protection structures. Therefore, emphasis is placed on minimizing potential ice damage using a variety of techniques.

(b) Leidersdorf, Gadd, and McDougal (1990) reviewed the performance aspects of three types of slope protection used for coastal projects related to petroleum activities in the Beaufort Sea. For water depths less than 2 m, sacrificial beaches appeared to function well. In water depths ranging from 7 m to 15 m, gravel-filled geotextile bags were able to withstand the larger wave forces, but they were susceptible to ice damage and required regular maintenance. Linked concrete mat armor (Leidersdorf, Gadd, and McDougal 1988; Gadd and Leidersdorf 1990) withstood both wave and ice loads in depths up to about 14 m. Mats were recommended for projects with a lengthy service life so that high initial capital costs would be offset by lower maintenance expenses.

Wuebben (1995) reviewed the effects of ice on riprap structures constructed along ice-prone waterways. This paper provided a good summary of successful riprap revetment design and construction practices based on actual field experience. Numerous useful references documenting ice effects on riprap are included in Wuebben's paper.

The following rules-of-thumb for arctic slope protection were given in Chen and Leidersdorf (1988) and summarized in PIANC (1992).

- Cover layers and underlayers should be strong enough to withstand local penetration by thick ice sheets.

- Smooth slopes without protrusions will reduce loads and allow the ice to ride up more easily without plucking out individual armor elements. (However, wave runup will be greater.)

- Flexible cover layers consisting of graded riprap may help absorb impacts by smaller ice blocks during wave action without appreciable damage. Sand bags are effective for structures with intended short service lives.

- Mild structure slopes are essential because they reduce the risk of ice penetration into the slope. Maximum slope 15 deg is recommended in the zone of ice attack.

- Compound slopes with a nearly horizontal berm above the swl provide a platform for piled-up ice in regions which experience frequent ride-up of ice sheets.

- Maximum ice loads will not occur at the same time as maximum expected wave loads. Therefore, slope design can consider each load condition separately.

VI-5-9. References

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